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THE DENSIFICATION OF LOOSE SAND USING COMPACTION PILES

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ABSTRACT

Two case histories of densification of loose sandy soils using compaction piles are analyzed. The sandy soils were improved to support heavily loaded structures using shallow foundations. The technique used was deep compaction through compaction piles driven with Franki -type equipment, and shallow compaction by vibratory plate. Results of the compaction processes are presented and discussed to provide guidance for future projects. The analysis includes distance from the compaction pile, initial relative density, time delay for results verification after compaction, and depth. The results of densification demonstrate the method is efficient.

INTRODUCTION

Soil improvement techniques are used extensively to avoid many of the settlement and stability problems associated with loose foundation soils. In the recent decades, compaction piles have gained wide acceptance for strengthening and reducing the compressibility of loose sands. This paper analyses a database of two case histories of densification of loose marine sandy soils using compaction piles in the state of Espirito Santo, Southeastern region of Brazil. Today the improved sandy soils support heavily loaded structures using shallow foundations. The technique used was deep compaction through compaction piles driven with Franki- type equipment, and shallow compaction by vibratory plate. Compaction piles densify primarily by displacement, besides some vibration occurring during driving. Densification evaluation is done by comparing in-situ measurements of sand penetration resistance conducted before and after the soil densification. The two case histories discussed show that in general compaction piles increase the penetration resistance of loose cohesionless soil in soundings. The analysis includes distance from the compaction pile, initial relative density, time delay for results verification after compaction, and depth.

DESCRIPTION OF SITE CONDITIONS

The two cases studies have several findings in common with the same geological origin. Both studies are located on the coast of the state of Espirito Santo, Southeastern region of Brazil. The region presents subsoil composed of marine sediments from the Holocene (quaternary age). The sand layers are of different relative densities probably due to stratifications occurring during rise and fall of ocean level (transgression and regression) through the ages.

Case Study No. 1. The first case is a 10-story apartment building located in the city of Vila Velha, Espirito Santo, Brazil. During the initial soil investigation for the building, standard penetration tests, SPT (efficiency of about 70% accordingly to most measurements in Brazil), were performed at several locations at the site. The soil profile primarily consisted of 1 meter of sand fill overlying 4 meters of clean sand ranging in density from very loose to very dense. At the base of these superficial layers there is approximately 0.5 meter of very loose organic clayey sand and then other layers that don't matter to this analysis. The ground water level at the test site was encountered at a depth of about 1.3 meters, during the investigation.

The density of the 4 meters of clean superficial sand varies considerably with both depth and lateral extent. Therefore, the test program site was divided into two distinct test sites, a very loose to very dense sandy site (area 1), and a very loose to loose sandy site (area 2). The two test sites were selected to verify the validity of the proposed loose sandy soil improvement technique.

The Standard Penetration test (SPT) N-values within the area 1 ranged from 4 to 10 blows per 300 mm, and the N-values within the area 2 ranged from 4 to 10 for the first 2 meters and below the 3 meters depth, the N-values ranged from 13 to 80. An additional subsurface exploration program was done using dynamic penetrometer tests. The dynamic penetrometer tests were performed in accordance with ISSMFE (1989). The results of standard penetration and dynamic penetrometer tests established that the locally clean sand required in-place densification to minimize differential settlements of shallow foundations. The densification technique was required to improve the soil to support conventional footings for a design bearing pressure of 0.3 MPa. The footings have a minimum width of 1 m and placed at an approximated depth of 1.3 meters below the ground surface.

Case Study No. 2. The second case study is an analysis of the foundation of a 6-story apartment building located in the city of Vitória, capital of the state Espirito Santo, Brazil. Several soil borings were performed to determine the subsurface conditions. It was found that the soil profile consisted of 13 meters of sand layers of different compactness degrees. The uppermost layer, approximately 6 meters thick, consisted of loose to medium dense sand. Underlying the sand layer there is approximately 7 meters of a medium to very dense sand. The ground water level at the test site was encountered at a depth of about 3.0 meters, during the investigation.

The results of standard penetration tests established that the top 6 meters of loose to medium dense sand required in place densification to minimize differential settlements of shallow foundations. A loose sandy soil improvement technique was required to support conventional footing for a design bearing pressure of approximately 0.3 MPa. The footings have a minimum width of 1 meter and placed at an approximated depth of 3.0 meters below the ground surface.

In order to use spread footings instead of deep foundations (which would pose problematic driving), in the case histories presented, the sands were densified to an acceptable relative density. The technique used was deep compaction through compaction piles driven with Franki-type equipment, and shallow compaction by vibratory plate. Compaction piles are not a deep foundation as in a typical pile. They are made of sand or sand/quarried stone mixture columns (the quarried stone is used for driveability reasons only) dynamically densifying the loose adjacent sands. The case histories were selected to verify the validity of the proposed loose sandy soil improvement technique.

COMPACTION PILE EXECUTION

The piles were made by using the usual Franki Process (Tomlinson, 1995). This method of compaction by piles is to drive a withdrawable tube with 20 kN hammer having a 6 meters drop falling within the tube. The pipe casing of 400 mm diameter is slightly longer than the length of the compaction pile. The blows of the ram strike a plug formed of sand and gravel or quarried stone at the base of the tube, which is open at both ends. The plug under the blows of the ram pulls the tube down by friction. After the desired depth has been reached, the plug is forced out by holding the tube while tamping. More sand (or sand/quarried stone mixture) is fed into the tube and tamped while the tube is withdrawn gradually. This forms a dense sand (or mixture of sand and quarried stone) pile with enlarged base and, if desired, enlarged shaft at chosen depths. Soil compression is produced during ramming by vibration, but principally by sidewise displacement of the soil.

The volume of loose sand replaced by injected sand (or sand/quarried stone mixture) is one of the most important factors in improving weak ground using compaction piles. The depth of base construction followed the recommendations made by Nordlund (1982) for conventional Franki-type piles. More details are provided by Bicalho et al. (2002). The specifications required, however, that the final spacing between compaction piles be determined by a field testing program performed at the site. For better field control of effectiveness, piles were driven first at the corners of the rectangular footing foundation, then at the center. Prior to driving additional compaction piles at intermediate points on each side, the increase in sand density was evaluated through penetration resistance of soundings after densification. The most common performance criterion is the comparison between the initial and final penetration resistance. Measurement of the effectiveness of compacting piling was made from the volume of sand added, together with Standard Penetration tests, SPT (yielding the SPT N-value), and Dynamic Penetrometer tests, PD (yielding a dynamic point resistance, q_d).

An approximate correlation between the Dutch cone penetration resistance, q_c, and the dynamic resistance parameter, q_d, proposed by Waschkowski (1983), is given by:

$q_d/q_c = 1$ (loose to medium dense sands) (1)

RESULTS AND ANALYSIS

Case Study No. 1. Penetrometer tests were carried out from the estimated depth of the footing foundation (i.e., -1.3 m) to the lower layer of medium to very dense sand (i.e., -5.5 m). Compaction piles spacing are generally in the range of 2 m to 3.2 m (5 to 8 pile diameters).

Penetrometer test results before (initial PD tests, q_{di}) and after (final PD tests, q_{df}) densification by compaction piles are shown Figures 1 and 2, where q_{di} is the initial dynamic resistance parameter for penetrometer tests, PD. Penetration resistance may serve as a measure of the amount of densification obtained. Test results indicate that compaction piles increased the penetration resistance of loose cohesionless soil in general. The wide scatter of the data, however, precluded fitting any meaningful curve through the points. A clear trend however is displayed: the penetration resistance increased with the depth. The intend of pile densification was focused on depths of 2.5 meters or more. Castello and Polido (1982) show that superficial cohesionless soils have low confinement, so when a pile is driven it displaces the soil to the ground instead of compacting it. The compaction piles are useless at shallow depths and other procedure has to be used. For the upper meter of sand the improvement was obtained with vibratory plates with mass up to 400 kg. Results of the PD tests after shallow compaction are not presented in this paper since the purpose herein is to discuss the compaction pile technique only, anyway the final PD tests results were $q_{df} > 15$ MPa.

Figures 3 and 4 present the relationship between the original penetration resistance, q_{di}, and the penetration resistance increase, $K_m = q_{df} / q_{di}$, in points. The tests results show a variation of q_{df} / q_{di} ratio of 2-7. From the data presented, it can be seen that compaction piles tend to compact the loosest soils the most, Figure 4. The results tend to produce a more homogeneous density, reducing future differential settlement. On the other hand, one can imagine that compaction piles are useless and maybe detrimental in sandy soils above a certain penetration resistance (or relative density).

The tests were carried out during the period 1-10 days after densification. The PD 1A test was carried at 40 days after densification. The test data clearly showed the penetration resistance increased with time after densification, Figures 1 and 3. Penetration resistance, strength, and stiffness of sands increase with time after densification. The mechanisms responsible for time effects are not known (Mitchell and Solymar, 1984). It is recommended, therefore, to further investigate the effect of time on compaction piles.

The PD 2A, PD 2B and PD 2C tests were carried out at points located at central area of the test side, area 2, where better results were expected. The test data, as shown in Figures 2 and 4, clearly confirmed this expectation. So most of the verification soundings were made at the periphery of the clusters of compaction piles. If the results were good there, they would be better at central points.

Furthermore, the test data clearly showed that the 0.5 meter very loose organic clayey sand had been virtually fully compressed by dynamic replacement and mixing with sand (as in a hydraulic fracture) after the densification process. The method of dynamic replacement and mixing of organic soils with sand was investigated by Lo et al. (1990).

Case Study No. 2. Standard Penetration tests, SPT, were carried out from the depth of 1.0 meter to the lower layer of medium to very dense sand. Compaction piles spacing are generally in the range of 1.4 m to 1.6 m (3.5 to 4 pile diameters).

SPT test results before (initial SPT N-value) and after (final SPT N-value) densification by compaction piles are shown in Figure 5, where the SPT N-value is the number of blows/300 mm. Penetration resistance was used as a measure of the amount of densification obtained. Test results indicate that in general compaction piles increase the penetration resistance of loose cohesionless soil. The wide scatter of the data, however, precluded fitting any meaningful curve through the points. A clear trend is displayed: the penetration resistance increased with the depth. The intend of pile densification was focused on depths of 4 meters or more.

Figure 6 presents the relationship between the original penetration resistance, initial SPT N-value, and the penetration resistance increase, K_m = Final SPT N-value / Initial SPT N-value, in points. The tests results show a variation of K_m of 2-15. As discussed in Case No. 1, the test data presented showed that compaction piles tend to compact the loosest soils the most. The test data also suggested that the penetration resistance increased when the compaction pile spacing was reduced from 2 -3.2 m to 1.4-1.6 m. It is recommended therefore, to further investigate the effect of pile spacing on compaction piles.

SUMMARY AND CONCLUSIONS

This study has shown the feasibility of using compaction piles executed with the Franki Process (Tomlinson, 1995) for the purpose of densifying loose cohesionless soil. Test results indicate that in general compaction piles of 400 mm diameter casings spaced on 1.4-3.2 m centers (3.5 to 8 pile diameters) can increase the penetration resistance of loose cohesionless soils by 2-15 times the initial penetration resistance. Loose cohesionless soils can be densified effectively using compaction piles.

From the data presented, it can be seen that compaction piles tend to compact the loosest soils the most. The results tend to produce a more homogeneous density, reducing future differential settlement. As the sand density increases the effectiveness of the method decreases.

The test data clearly showed the penetration resistance, which serve as a measure of the amount of densification obtained, increased with time after densification. If the penetration resistance is used as a basic for quality control of densification, then values measured at early times will be conservative.

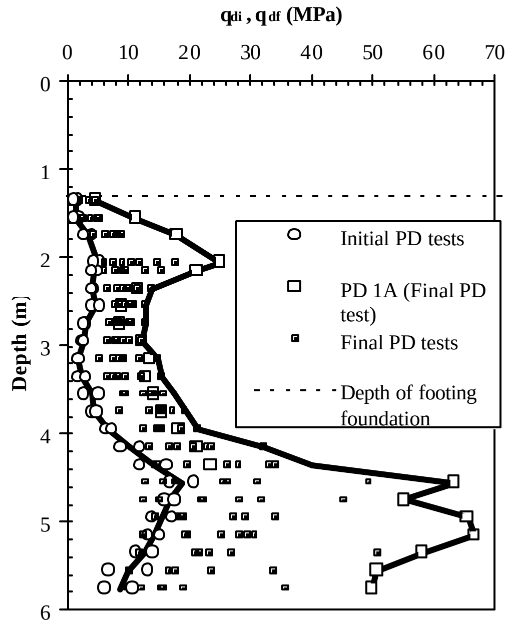

Fig. 1. Penetrometer test results in area 1 before and after densification by compaction piles (Case study No. 1)

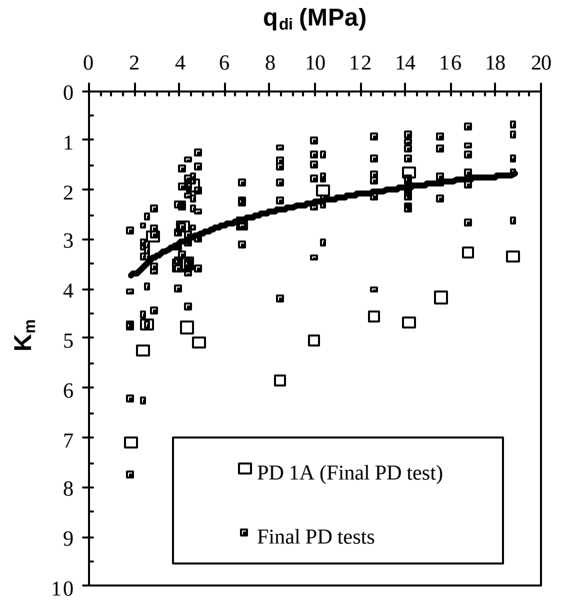

Fig. 3. Variation of q_{df}/q_{di} ratio with initial Penetrometer tests (area 1) (Case study No. 1)

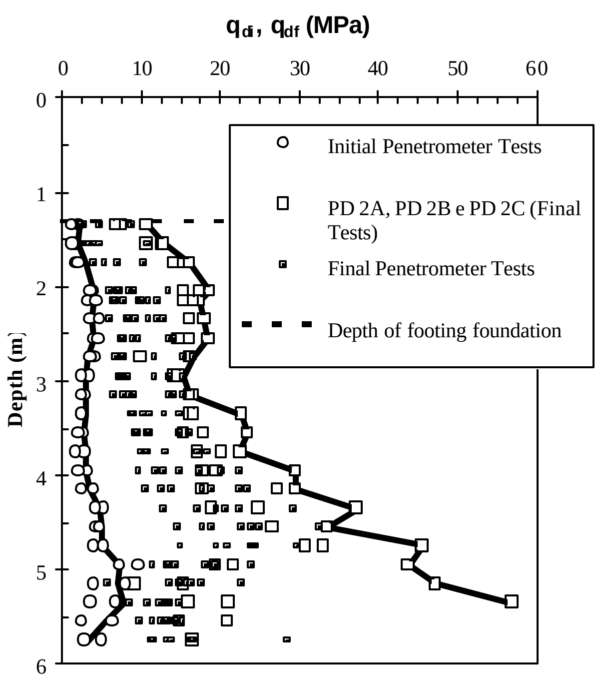

Fig. 2. Penetrometer test results in area 2 before and after densification by compaction piles (Case study No. 1)

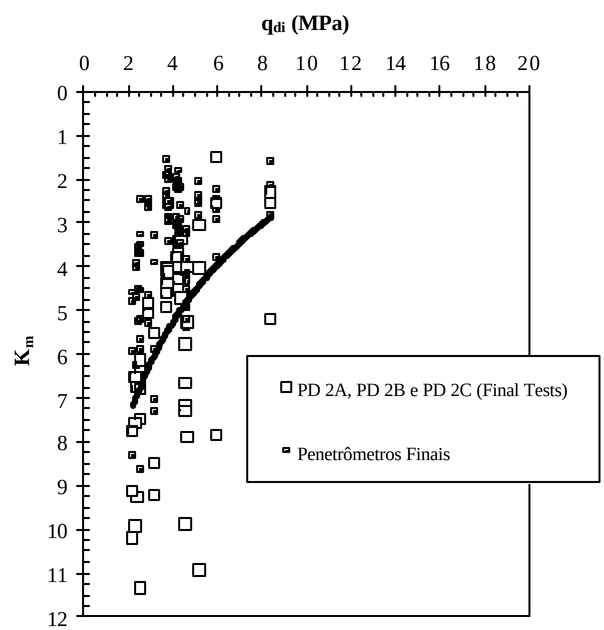

Fig. 4. Variation of q_{df}/q_{di} ratio with initial Penetrometer tests (area 2) (Case Study No. 1)

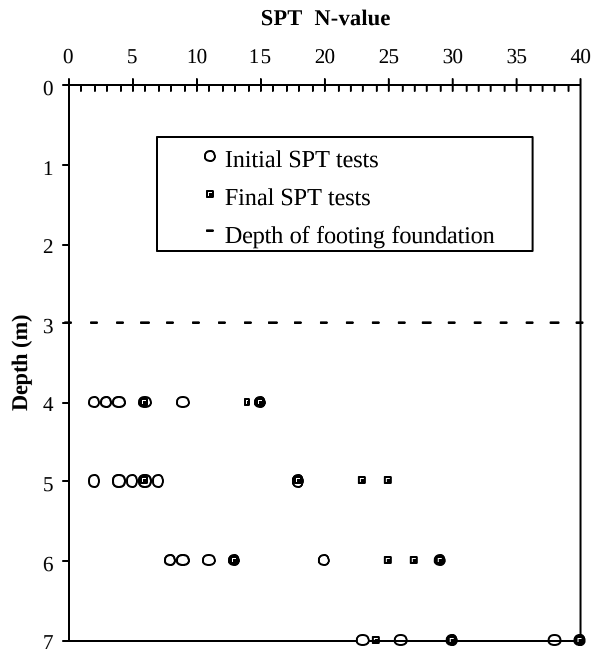

Fig. 5. Standard penetration test results before and after densification by compaction piles (Case study No. 2)

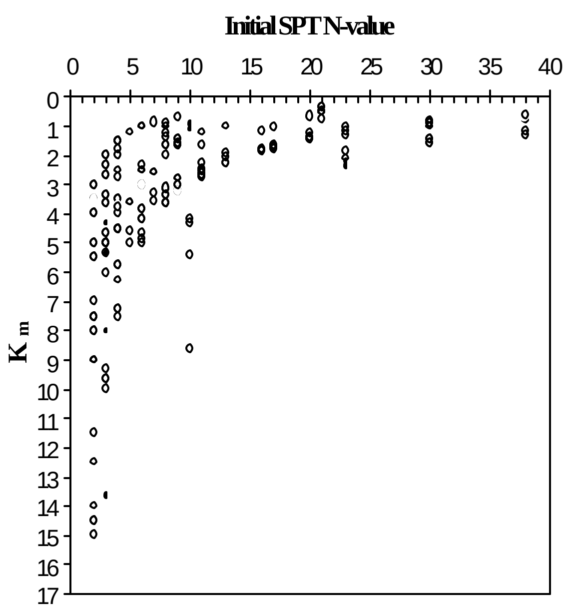

Fig. 6. Variation of K_m ratio with initial Standard penetration tests (Case Study No. 2)

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